WJE | ENGINEERS ARCHITECTS

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**VIA E-MAIL (jhartnett@faegre.com)**

December 11, 2009

Mr. James Hartnett, Esq.
Faegre & Benson, LLP
2200 Wells Fargo Center
90 South Seventh Street
Minneapolis, MN 55402

Re: Target Riverdale Plaza Precast Panel Failure
WJE No. PC09.1981

Dear Mr. Hartnett:

At the request of Faegre & Benson LLP and Target Corporation, WJE Engineers & Architects, P.C. (WJE) has performed a limited investigation to opine on the cause of the precast panel connection failures at the Target store T1798 located in Bronx, NY. This letter serves as our report on this matter.

## Background and Description of Structure

The Target Store is located in the Riverdale Plaza which is located at 300 West Kingsbridge Road (40 West 225th Street), Bronx, NY. The large structure that houses the Target store also has an additional retail space (K1). The Target portion of the building in plan view is comprised of two rectangular sections. The main section has outside dimensions of approximately 297 ft. east/west by 477 ft. north/south (Figure 1). The other section, which houses receiving and warehouse spaces, extends from the southeast corner of the main section and has outside plan dimensions of about 180 ft. north/south by 109 ft. east/west (Figure 2). The K1 retail portion of the building is rectangular in plan with a truncated southwest corner. It has outside dimensions of approximately 227 ft east/west and 308 ft. north/south (Figure 3). All of these sections are separated from each other by expansion joints as shown in figure 4. The building is a 1 story structure with a mezzanine level.

The roof deck of the Target/K1 structure serves as parking for the stores. There are 2 ramps to access the roof deck. The Target ramp on the north side of the building has direct access off of West Kingsbridge Road. The K1 ramp on the west side of the building has access from the grade level retail parking area. There are also bulkhead structures on the roof that serve as entryways into both the Target and K1 portions of the building as well as fire escapes. (Figure 5 and 6).

The building has a structural steel frame and composite lightweight concrete roof slab on steel deck. The steel frame supports architectural precast concrete panels that make up most of the building's façade. The remainder of the building's façade (i.e. Target entry, K1 retail entry, and bulkhead entries) is composed of a glass curtain wall systems and screen wall systems covering selected precast panels and extending above the roof at some locations. Fire escape bulkheads are typically made of brick masonry.

The building was designed by Greenberg Farrow Architects (GFA). Goldreich Engineering P.C. (GE). GE was the structural engineer of record. Construction began in January 2003 and the store opened around July 2004.



**Exhibit B**

In December 2008, a vehicle struck one of the precast concrete wall panels near the K1 ramp at the southwest corner of the building. The vehicle strike caused the panels in the area to tilt outward. When the precast panel connections were exposed at the struck panel, they were found to be completely severed. Adjacent precast panel connections were also exposed and found to be similarly damaged.

Prior to this event, Target also reported that many of the lightpole bases had failed requiring a repair program to the replace the concrete bases.

In January 2009, Target hired American Engineering Testing Inc, (AET) to perform a condition assessment of additional precast connections on all elevations of the Target building. AET's investigation revealed that 65 of the 75 "type 62" connections reviewed were found to be severely damaged, to the point where they could no longer provide any resistance to lateral loads. Figure 7, from AET's report dated February 12, 2009, shows the location of the connections inspected and their conditions.

Connection "type 62" is a slotted insert connection that is cast into the back of the precast panel. It is manufactured by PSA. To provide a lateral connection back to the steel frame, a threaded rod/weld plate is welded to the top flange of the steel spandrel beam and attached to the insert by a nut behind the slot (Figure 8).

Following the AET condition assessment, Peter Cicuto Services Inc. (PCS) issued a letter on February 12, 2009. PCS reviewed the conditions at the precast panel connections on behalf of Global Precast, who was the precaster for the original construction. PSC's letter states that the damage was caused from either snow removal or expansion of the concrete topping slab.

On February 23, 2009, Target issued a set of drawings for an extensive inspection and repair program of the precast panel connections. Inspection of most perimeter connections was performed and documented by Schimenti Construction. Figure 9 shows the results of the connection inspections performed. 151 panels were inspected and of the 314 top panel connections, 63% of them were found to be broken.

On August 17, 2009 Thornton Tomasetti (TT) issued a report stating that all damage to the precast panels was directly related to snow plow damage.

WJE was asked by Faegre & Benson LLP, representing Target, to review the condition of the precast panel connections and opine on the cause of the connection failure. We were also asked to review and opine regarding the PCS and TT reports.

## WJE Observations

WJE reviewed GFA's bulletin 4 drawings dated October 9, 2002 for details pertaining to the roof slab and precast panel connections. The roof slab consists of a 5 ¼ in. lightweight concrete slab on top of a composite 2 in. deep, 18 gauge steel deck. The typical structural detail for the slab/precast wall panel interface is shown in detail 3/S-8 (Figure 10). In this detail, a 1 in. gap is shown between the edge of the structural slab and the face of the precast panel. No details for the roof buildup or topping slab are included in the structural drawings.

The architectural drawings show the roof build-up above the 5 ¼ in. structural slab. A waterproofing membrane is shown directly adhered to the structural slab. Above the membrane, in order, is a ¾ in. protection board, 4 in. of rigid insulation, and a 4 in. thick normal weight concrete topping slab. At perimeters, the topping slab thickens to 10 in. to serve as a curb. Detail 2/A-410 and 3/A-410 show the typical details for how the concrete topping slab terminates at the precast panels and CMU bulkhead walls respectively (Figure 11 and 12). The detail for the topping slab shows a ½ in. joint filler between the 4 in. topping slab and 10 in. curb. The detail then calls for a maximum 1 in. gap between the back edge of the curb and the face of the precast panel. No gap is specified around the guardrail post that penetrates though the curb/topping slab. This guardrail post extends down thru the structural slab and is welded directly to the top of the steel spandrel beam. Also, the architectural drawings show a 1 1/2 in. space between the back of the pour stop for the structural slab and the inside face of the precast panel (shown as 1 in. on the structural drawings).

On June 26, 2009 John Cocca of WJE met with Brad Koland of Target Corporation to review the conditions at site. At the time of our visit, no connections were exposed for our review. The contractor had just begun work on the north elevation and was finishing work on the south elevation.

At the time of our visit, all precast panels along the north elevation had been shored with a tieback to the topping slab. WJE observed that every guardrail post along the north elevation was leaning outward (towards the north). There were wide cracks and spalls in the concrete around the posts where it penetrated the topping slab. In general, the tension cables between the posts were found to be intact and in good condition (Figure 13). Also, most of the light gauge metal frames and stainless steel mesh were intact and in good condition. There was only one location at the northeast corner where the mesh was missing and plywood was covering the opening.

Cracking and displacements of the masonry at the bulkhead structures at the northeast corner and southwest corner were observed (Figures 14-16). Spalling of the exposed precast connections at the northwest corner behind the stainless steel mesh structure was also observed (Figure 17).

WJE observed areas within the building along the north elevation (column line 18 between C and D) where the spandrel beam that supports the precast panels was exposed from the interior. Fireproofing was found to have spalled and cracked along the bottom flange. Cracking was also observed in the perpendicular CMU wall (Figure 18).

On August 19, 2009 Andy Osborn and John Cocca of WJE returned to the site to meet with Brad Koland and review the exposed precast connections at the north elevation along column line 18 between B-H. Two modes of failure were observed by WJE. The first failure mode was failure of the concrete around the embedded anchor causing the embedded anchor to pull out (Figure 19). The second failure mode was failure of the steel embedded anchors where the lips of the steel on each side of the slot yielded, allowing the nut to pull out of the slot (Figure 20). Brad Koland reported a third type of failure in which the threaded rod portion of the connection has failed. WJE did not directly observe one of these failures at the time of our visit.

WJE verified the existing roof build-up which varied slightly from the design drawings. WJE observed the 5- 1/4 in. structural slab on metal deck with a waterproofing membrane adhered to the structural deck. On top of the waterproofing membrane, WJE observed a drainage mat covered with a filter fabric

followed by two layers of 2 in. rigid insulation (4 in. total) followed by 4 in. concrete topping slab and 6 in. curb at the edges (Figure 21). At the northeast corner, WJE observed a location where an opening in the topping slab and curb was made adjacent to a section of curb/slab remaining in place. The section remaining in place could be viewed from the side (Figure 22). An approximate 1 1/2 in. gap was observed between the edge of the structural slab and the face of the precast panel. The waterproofing membrane turned up the face of the precast panel approximately 1 ft. as shown in the design drawings and terminated with a termination bar. The filter fabric and drainage mat terminated at the precast panel and did not turn up the wall (Figure 23). The construction largely comports with the architectural drawings except as follows: A drainage mat and filter fabric were used in lieu of the protection board shown in the drawings and two layers of Extruded Expanded Polystyrene rigid insulation (DOW blue board) were used. The concrete at the edges was cast up against the precast panels with a piece of fiberboard between the precast panel and edge of slab. The curb was then cast on top of the topping slab as opposed to being cast monolithically next to the edge of the topping slab as shown in the drawings. Also, the steel curb edge specified in the drawings was not installed.

From this view, WJE observed that there was no gap cast between the edge of the topping slab/curb and face of the precast panel. Only the waterproofing membrane and an approximately 1/2 in. thick piece of fiberboard separated the slab edge from the precast panel face. Furthermore, a lateral displacement between the two layers of rigid insulation was observed.

Finally, WJE observed the interiors and exteriors of the remaining bulkhead structures. Most of the bulkhead interiors were finished with drywall on the inside preventing WJE from viewing the condition of the CMU walls at the topping slab elevation. However the northeast stair tower bulkhead structure and south stair bulkhead had cracking and displacement of the interior CMU at the elevation of the topping slab (Figure 24 and 25). Furthermore, the northeast stair tower's western CMU bulkhead wall was found to be out of plumb. Cracking of the exterior brick veneer was also observed. Both repaired cracks as well as new cracks were observed (Figure 26 and 27).

## Review of Peter Cicuto Services and Thornton Tomasetti Reports

WJE has reviewed both the report issued by PCS on February 12, 2009 and the report issued by TT on August 17, 2009. The PCS report concludes that the damage of the precast panel connections was caused by either snow removal or thermal expansion. The TT report concludes that the damage of the precast connections was caused solely by snow removal.

In TT's observations, they state that numerous guardrail tension cables were missing. WJE only found select locations where single cables were missing. There are a total of 6 cables strung from post to post. WJE did not find any locations where all six cables were missing. Furthermore, WJE only observed one location were the stainless steel screen mesh was completely missing. TT also claims that the remaining screens were bent outward; WJE did not observe this condition. The only screen that appeared to be bent outward was the screen directly adjacent to the missing screen (see TT photo 5). Remaining screens appeared to be in good condition.

Similar to WJE's observations, TT states that they observed "*no measurable joint between the topping slab and the face of the precast concrete at locations where the curb has been removed and the concrete topping remained.*" Had the precast panel been impacted during snow removal, a measureable gap at this location would be present. After the panel was impacted, it would displace outward and the connection

would be damaged leaving a measureable gap between the slab edge and face of the precast panel. Had the slab thermally expanded, the expanding slab would push against the face of the precast panel displacing the precast panel damaging the connection and leaving no measureable gap between the face of the precast panel and the slab edge.

In TT's thermal expansion discussion section they reference calculations produced by Gennady Saratovsky of GE dated August 5, 2009.

In their discussion, TT claims "*The absence of compressed sealant indicates that no forces due to thermal expansion were present at the boundary between the concrete curb and precast panel.*" As noted in TT's observations, no joint was measurable between the topping slab and face of the precast panel; therefore as the slab thermally expands there is no open joint for the sealant to be compressed in.

In the thermal expansion calculations provided by GE they use a joint size of 1-1/2 in. between the slab and precast panel. This joint size is correct for the structural slab as observed by WJE and shown on the architectural drawings; however this joint size is incorrect for the topping slab as observed by both WJE and TT. The only thing separating the topping slab from the precast panel is the waterproofing membrane and asphalt filler. TT's report has these items being 6/32 in. and 1/2 in. respectively totaling 11/16 in. not the 1 1/2 in. reported by TT. All these items are stiff and relatively uncompressible. They do not constitute a "gap" in the sense of an air gap that is compressible.

In GE's calculations they show the roof buildup to be a 4 in. topping slab on top of a rubber membrane on top of a structural slab. This roof buildup description is incorrect compared to what is shown on the drawings and was observed in the field. GE is also assuming that the plane of lateral movement is between the concrete slab and rubber membrane. This is also incorrect in that the topping slab only touches the insulation board and not the rubber membrane. Based on WJE's observations and data provided by Dow Corning, the plane of lateral movement is between the two layers of insulation. The coefficient of friction between concrete and Styrofoam ranges from 0.65-0.9 whereas the coefficient of friction between two layers of Styrofoam is 0.6 according to Dow Corning.

Both the GE and PCS reports calculate the thermal expansion of the topping slab. PCS's report over calculates the thermal expansion by using a length of 600 ft. WJE reviewed the structural drawings and found the critical length in the north/south direction to be along column line B. The topping spans 476 ft. 8 in. Using a temperature differential of 60 degrees Fahrenheit a total expansion of 1.9 in. or 0.94 in. at each end of the slab is calculated. WJE's values appear to be in line with those calculated by GE. In GE's calculations friction forces are calculated. Since the coefficient of friction of the Styrofoam is 0.6, GE's value for frictional resistance of 4050 kips would not change. However, since there is not a 1-1/2 in. gap between the slab edge and face of panel, GE's thermal expansion forces are incorrect. If you use the space between the panel and slab as being 11/16 in. and conservatively assume that the material in the space (membrane and fiberboard) is capable of being compressed by 50% the thermal expansion force becomes approximately 11,600 kips. This value exceeds the frictional resistance between the layers of insulation and therefore, contrary to the TT report, the thermal expansion of the slab would induce forces on the precast panel connections.

In the TT reports snow removal discussions; they again refer to a set of calculations performed by GE. These calculations heavily depend on assumptions including what type of vehicles were used, how much

they weigh, how fast they were traveling when they supposedly hit the snow bank in front of the precast panels, the shape of the snow bank, the dimensions of the snow bank, the density of snow and the kinetic energy losses. The calculations produced by GE calculate the kinetic energy of the moving mass (snow + vehicle) and convert it to strain energy of the structure (panel connection and panel bending). In the calculations, GE only analyzes half of the precast panel and a single connection. Typical snow plows are approximately 8 ft. therefore the entire vehicle load cannot be transferred into only one of the two connections, force redistribution in the precast panel would also allow both connections to resist the impact load. While we don't agree with the modeling assumptions, if one uses the procedure provided by GE for the entire 8.5 ft. panel as installed, with two connections, the kinetic energy of the moving mass changes from 537 ft-lbs to 671 ft.-lbs. The spring strain energy or connection capacity (293 ft-lbs) is then multiplied by two since two connections are provided. Finally, the panel bending value increase from 146 ft-lbs to 275 ft-lbs since the moment of inertia of the entire panel is used. Therefore, the demand is 671 ft-lbs and the capacity is 861 ft-lbs. resulting in the connections being able to resist the forces from the snow plow. These calculations also assume 8 in. thick precast panels whereas the panels along the north elevation are 10 in. thick. This thicker panel would further increase the moment of inertia therefore further increasing the capacity value above. Also, the equations used for calculating the strength of the inserts are based on the ultimate strength of the insert and using linear elastic theory. The behavior at the ultimate load, when the steel lips begin to yield, would be plastic. Therefore the calculated strain energy per insert is under estimated.

TT acknowledges that panels in corners cannot be impacted by snowplows. The TT report postulates that the failed connections of precast panels that are protected from a direct impact are the result of the impact forces of adjacent panels being transferred through the inter-panel connections. The inter panel connection shown in Figure 28 consist of a plate crossing the panel joint with a single bolt on either edge of the panel. The tensile capacity of this bolted panel edge connection is about 4000 lbs, which is less than that of the lateral panel insert connection (type 62). Therefore, the panel-to-panel connection will fail long before sufficient load is transferred to fail the slotted insert connection. Furthermore, if forces were being transferred from an impacted panel to an adjacent panel, twisting of the panels would be observed at the panel joints. WJE did not find observe any panels that appeared to be twisted or out of alignment relative to the panel directly adjacent to it.

## Discussion and Conclusions

Based on review of the conditions and provided documents, it is our opinion that the damaged precast panel connections are the result of thermal expansion of the topping slab.

It is WJE's opinion that an error in the design drawings on detail 2/A-410 (Figure 8) is the cause of the damages. Had the dimension between the edge of the curb and face of the precast panel been dimensioned as 1 in. minimum instead of 1 in. maximum, a total of 2 in. would have been provided for thermal expansion of the topping slab. By labeling the dimension 1 in. maximum, the contractor was not obligated to leave any air gap between the edge of the topping slab and the face of the panel. Also, no gap was specified between the guardrail posts and curb, the curb and bulkhead walls or the slab and light pole bases. The drawings did however acknowledge a 1/2 in. compressible filler to be installed between the topping slab and the curb. The contractor did not install this compressible filler in between the curb and topping slab. Fiberboard was instead provided in-between the precast panel and topping slab. Even if the compressible filler was provided per the drawings, it would not have provided enough allowable movement to prevent damage of the precast panel connections.

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Further evidence of slab expansion is provided by the modified calculations above and the visual evidence of no gap between the topping slab and face of the precast panel observed by both WJE and TT. Additional evidence of slab expansion is provided inside the bulkheads where the expanding slab is displacing the CMU walls on the interior at the topping slab elevation and at the guardrail posts where the expanding slab has bent every post in the direction of the slab movement. The quantity of broken connections and the fact that connections have been damaged on all four sides of the building indicates that the expanding slab is causing the failures. As shown in Figure 9, most of the perimeter precast panels connections have failed. At areas where the precast panels have short slab spans due to their close proximity to expansion joints (southeast corner and east ramp) none of the connections were found to be damaged. Removing these panels from the total count, there are 109 perimeter panels and 230 top connections of which 86% were found to have failed.

The snow plow theory is refuted by both the modified calculations discussed above and the fact that no visual impact damage was observed on any precast panels or guardrail posts. There were no locations observed where all of the tension cables between the guardrail posts were missing. Had snow from either stacking or impact caused one of these cables to break, one would expect that the remaining 5 would have also been damaged. Furthermore, it is of our opinion that had snow been piled against the stainless steel mesh screens, the light gage metal frame connections would have failed before the precast panel connections. WJE did not observe any distress at the metal frames supporting the stainless steel mesh. Vehicle impacts would not result in damages to precast panels near the corners of the deck. Any forces transferred to corner panels through the panel alignment connections would result in twisted panels and failed alignment connections. No such misalignments or failures were observed. Finally, a panel insert connection failure caused by impact would leave an air gap between the panel and the edge of the topping slab and torn sealant. No such air gap or torn sealant was observed.

Please call if you have any questions regarding this report.

Sincerely,
**WJE ENGINEERS & ARCHITECTS, P.C.**

John Cocca
Project Associate

Andrew Osborn, P.E.
Senior Principal

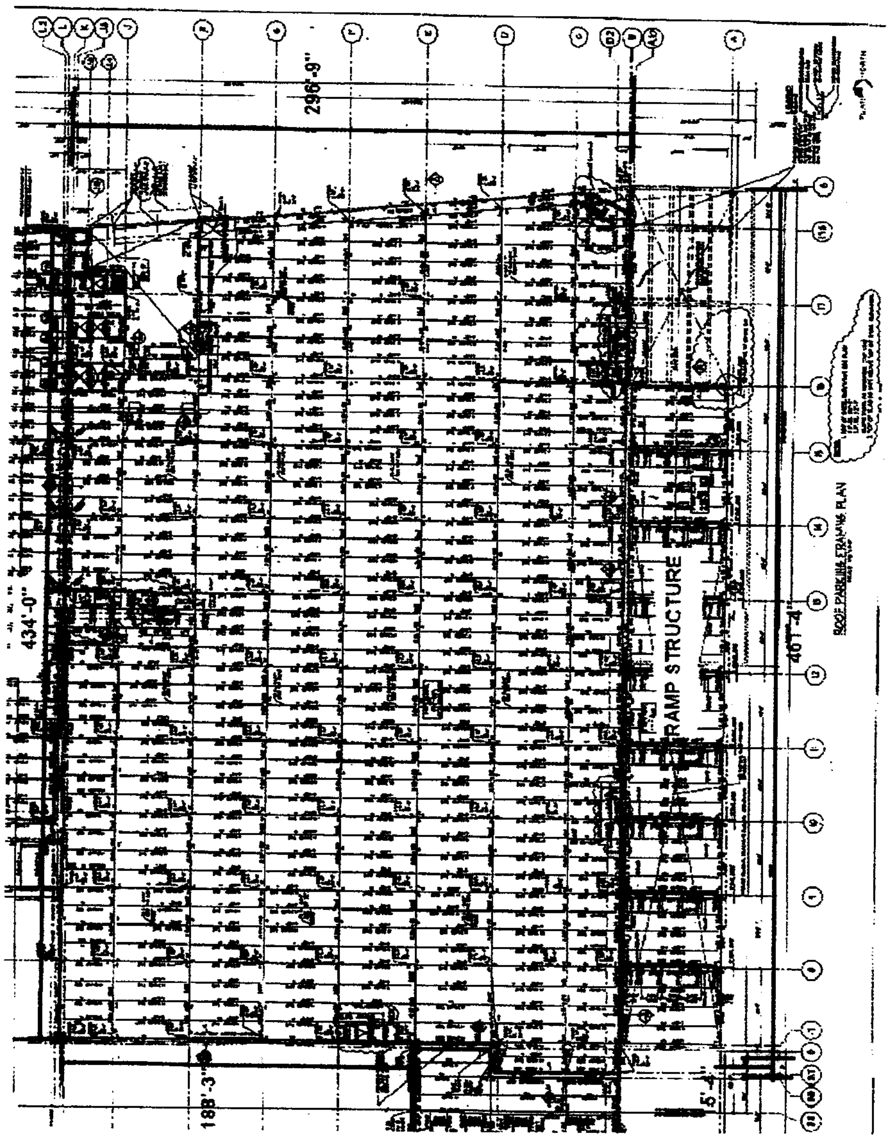


*Figure 1- Target Plan View - roof framing plan*

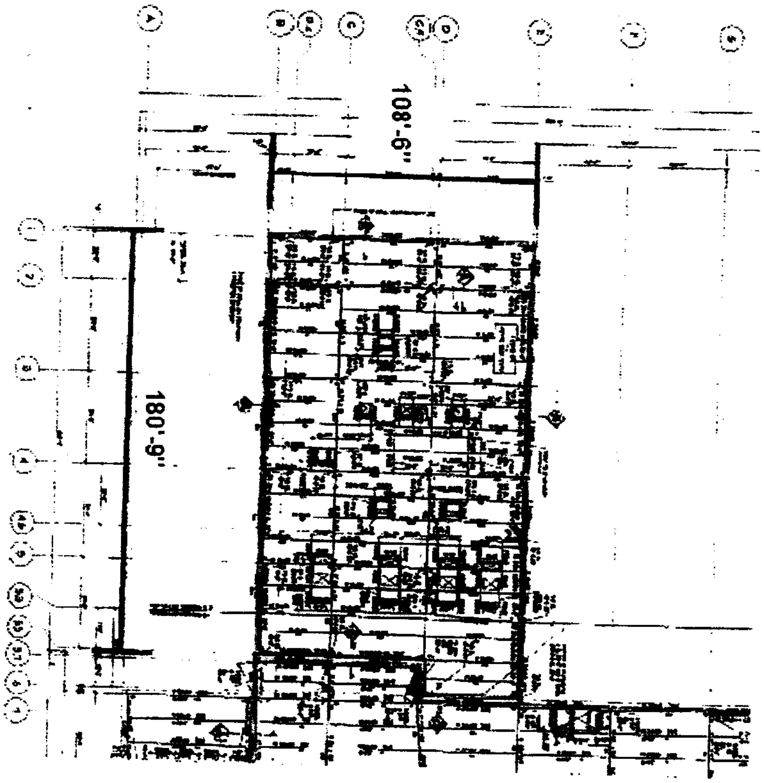


*Figure 2- Target Warehouse Plan View*

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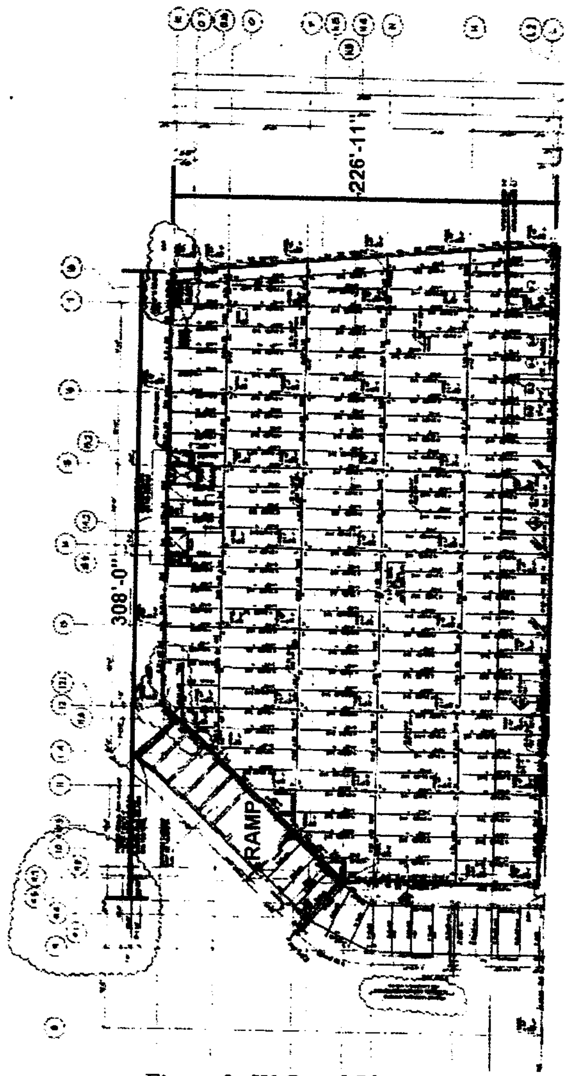


*Figure 3- K1 Retail Plan View*

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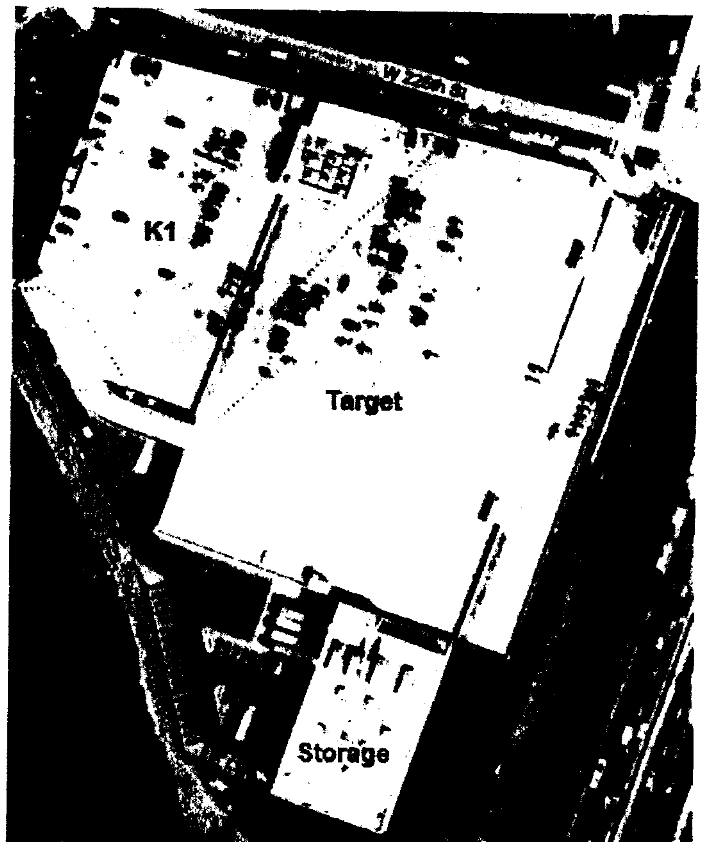


*Figure 4- Plan View of Store Showing Expansion Joints (Red Lines)*

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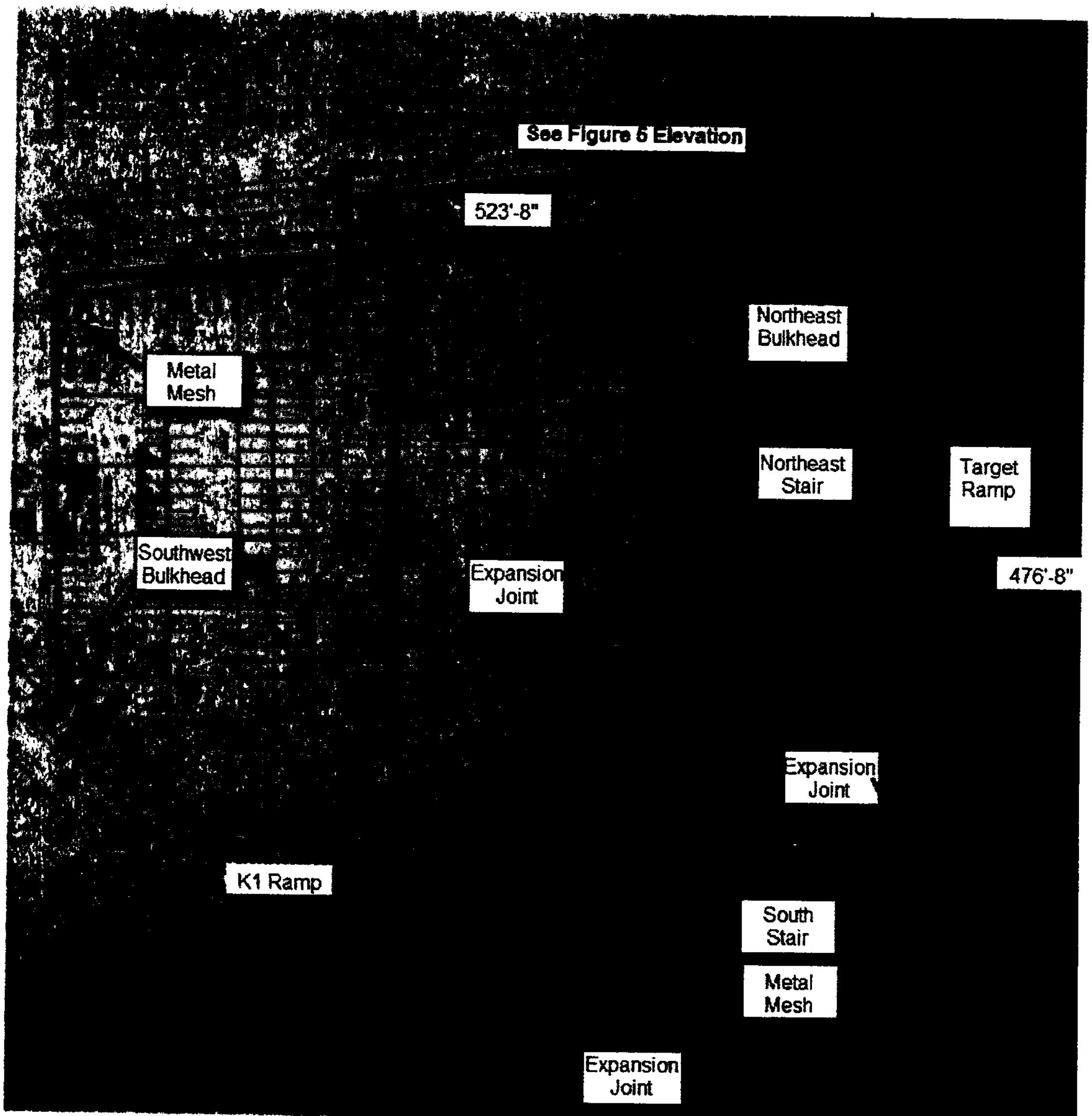


*Figure 5- Overall Roof Plan View*

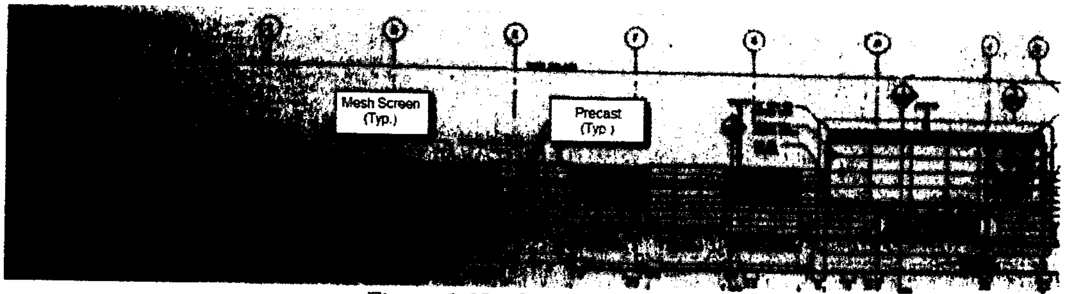


*Figure 6- North Elevation View*

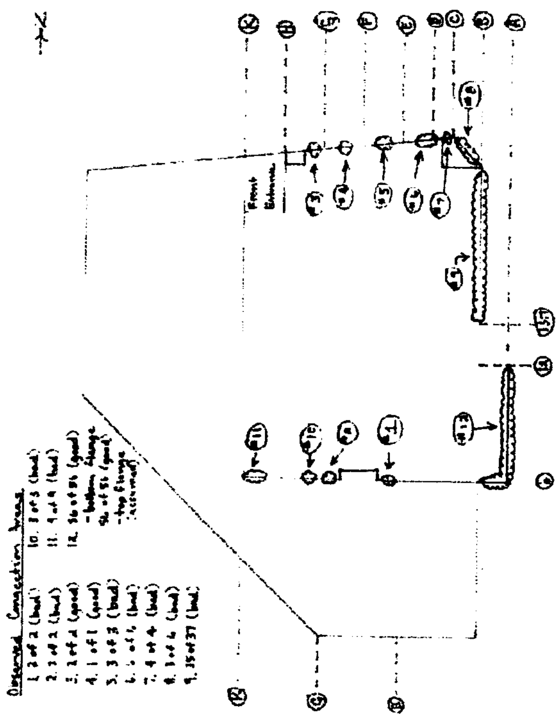

*Figure 7- Location and Quantity of Broken Connections from AET Report*

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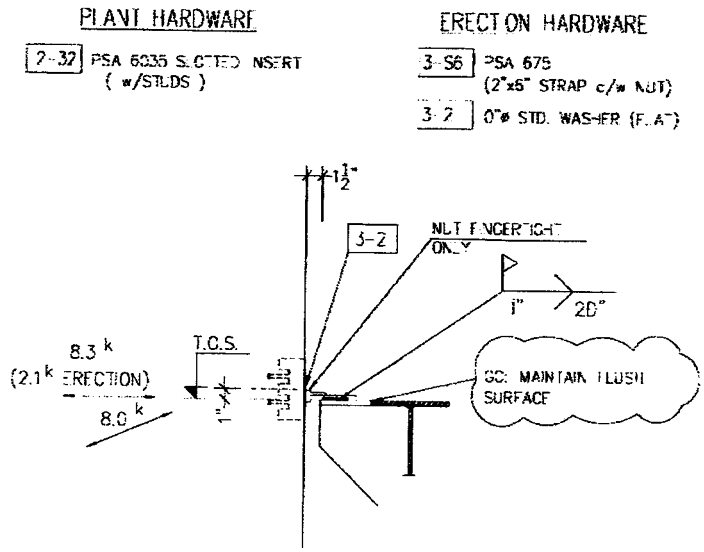


*Figure 8- Typical Connection "Type 62"*

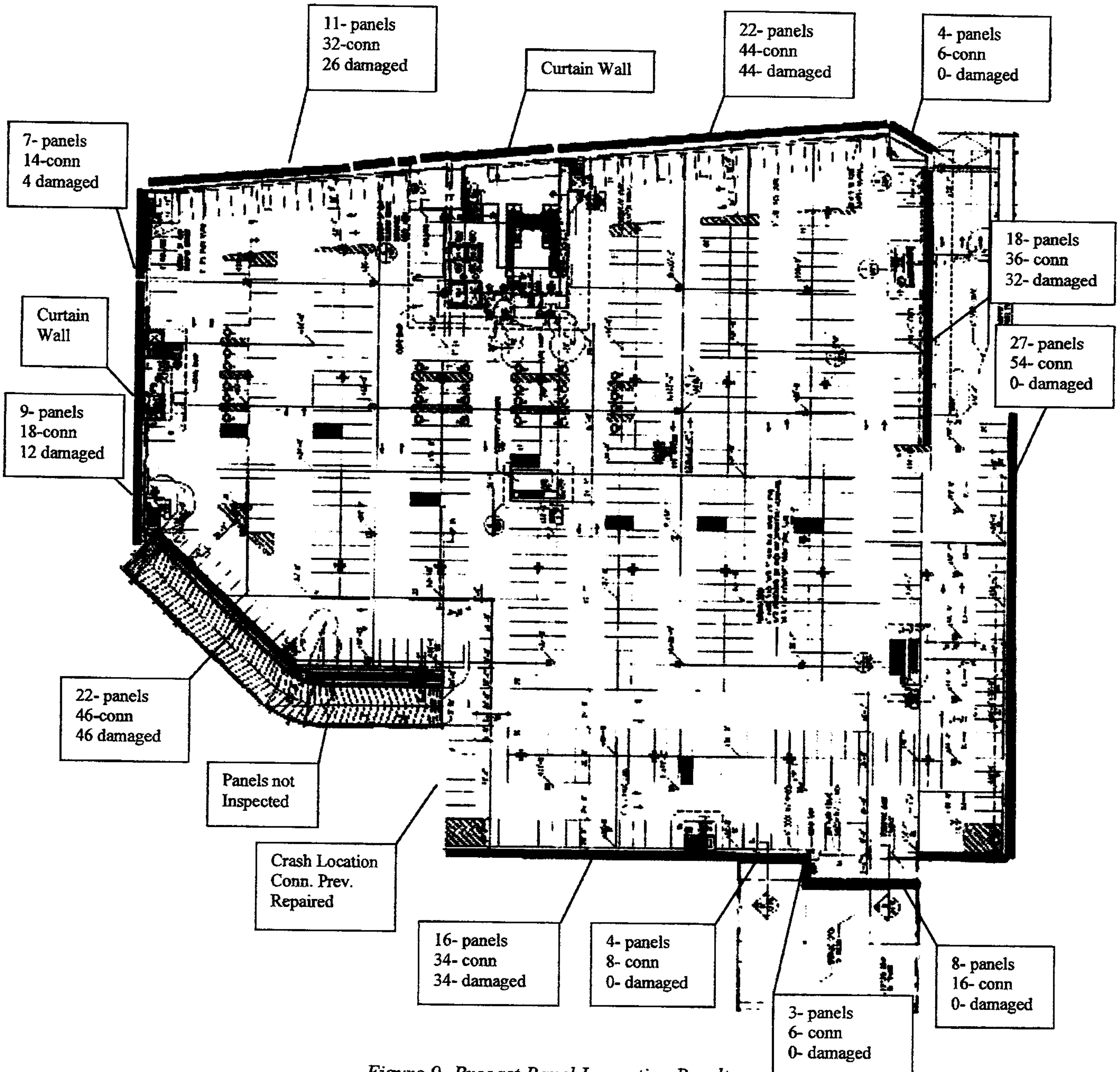


*Figure 9- Precast Panel Inspection Results*

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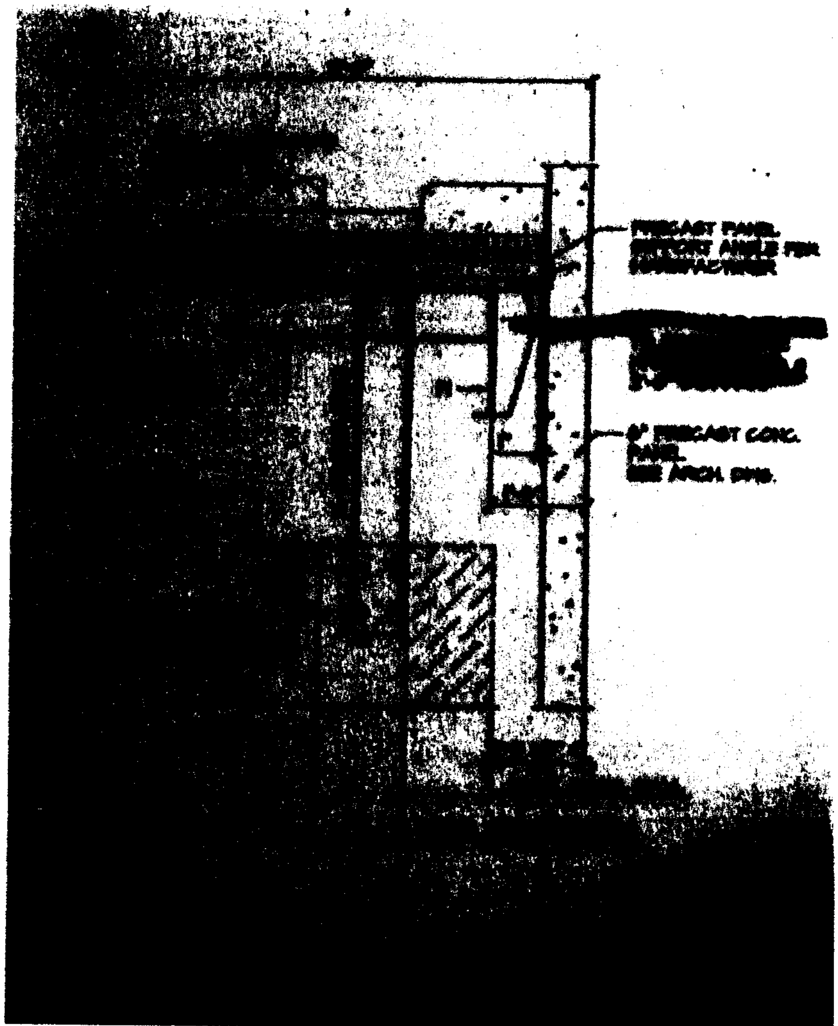

*Figure 10- Detail 3/S-8 from GFA Documents*

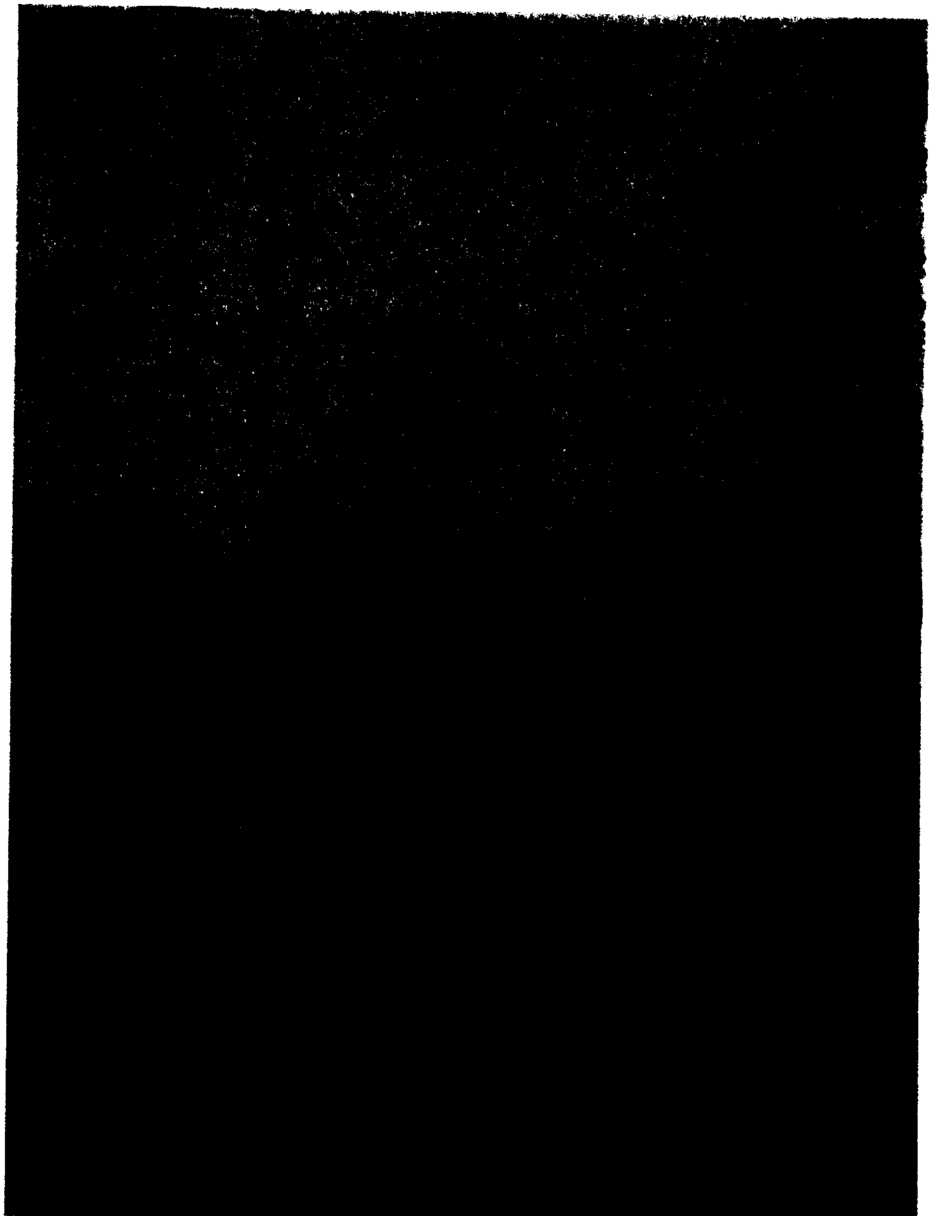

*Figure 11- Detail 2/A-410 from GFA Documents*

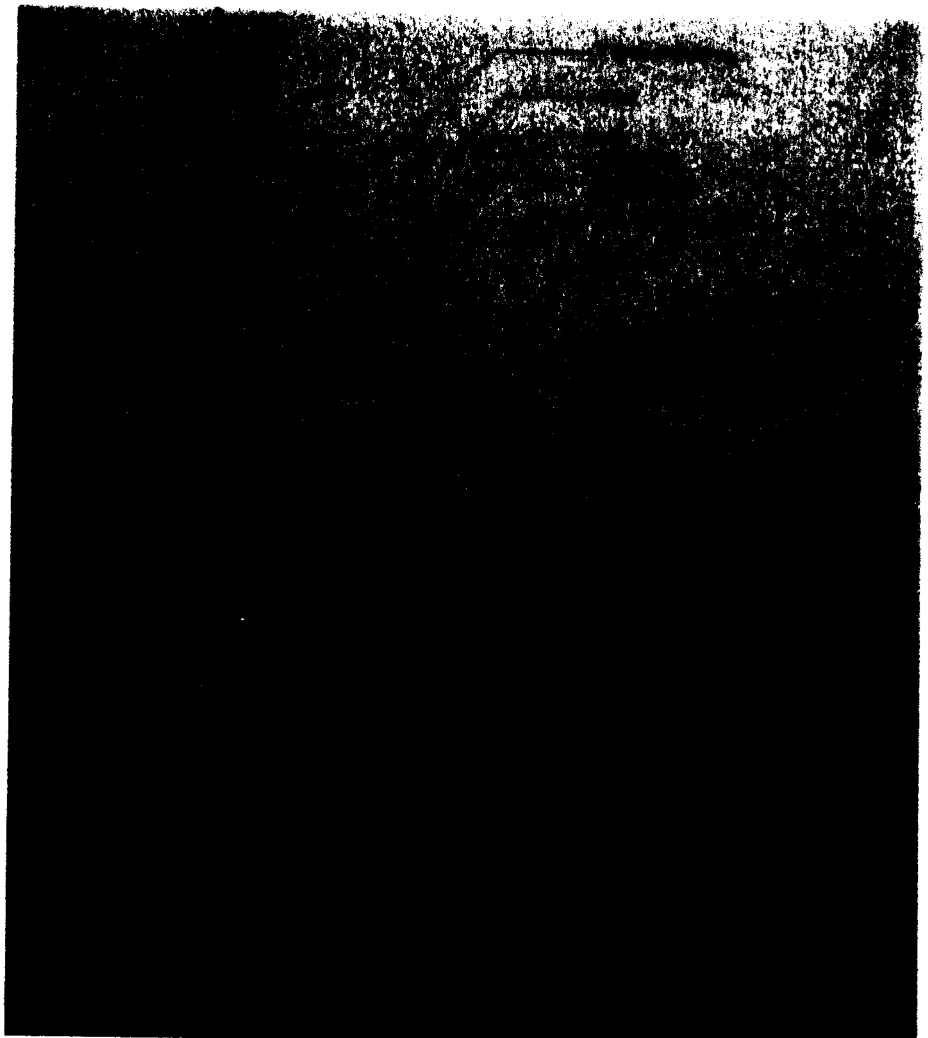

*Figure 12- Detail 3/A-410 from GFA Documents*

*Figure 13- Precast Panels along North Elevation between Northeast Corner and Target Entry. Note intact barrier cables and metal screens.*

*Figure 14- Cracking at Northeast Bulkhead*

*Figure 15- Cracking and Displacements at Northeast Bulkhead*

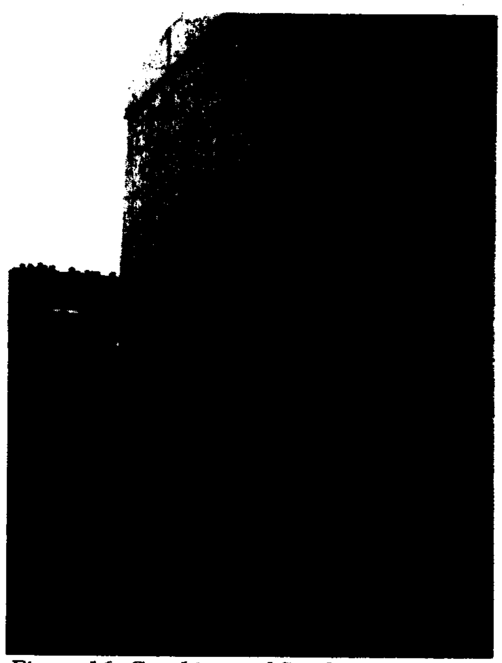

*Figure 16- Cracking and Southwest Bulkhead*

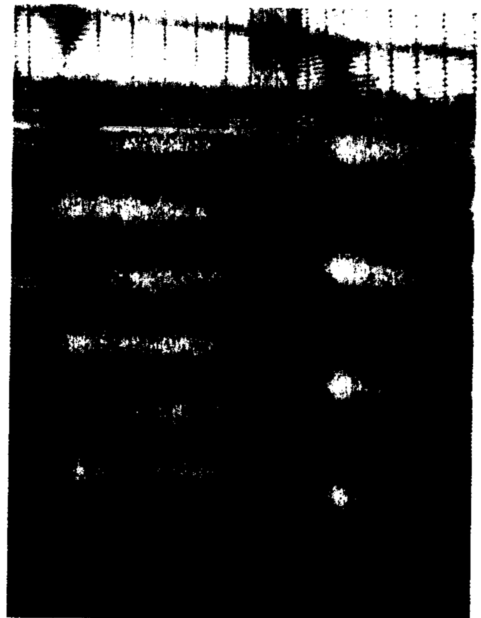

*Figure17- Spalling of Exposed Precast Connections at Northwest Corner*

*Figure 18- Cracking of CMU wall / Spalled and Cracked Fireproofing*

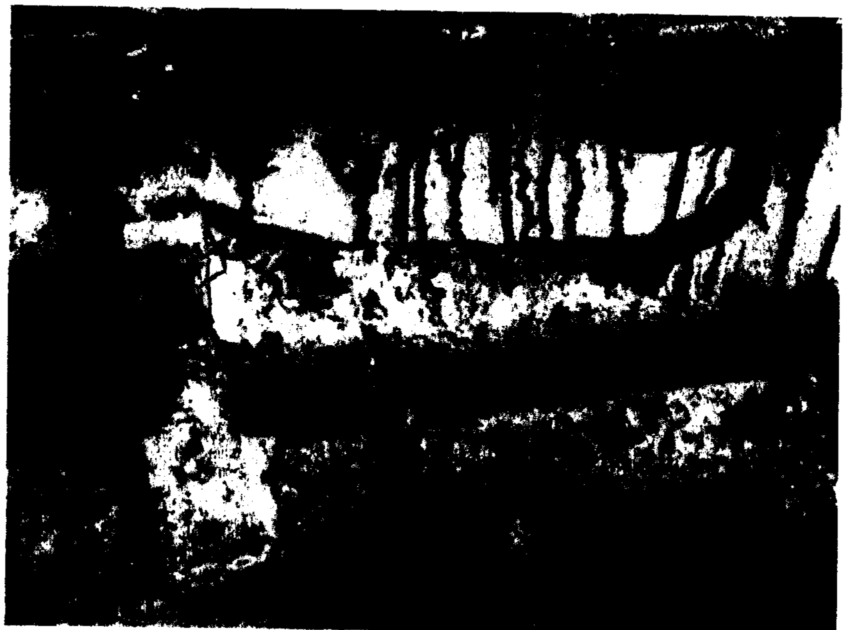

*Figure 19- Precast Panel Connection Failure - Failure of Concrete*

*Figure 20- Precast Panel Connection Failure - Failure of Embedded Anchor Lips on either side of slot.*

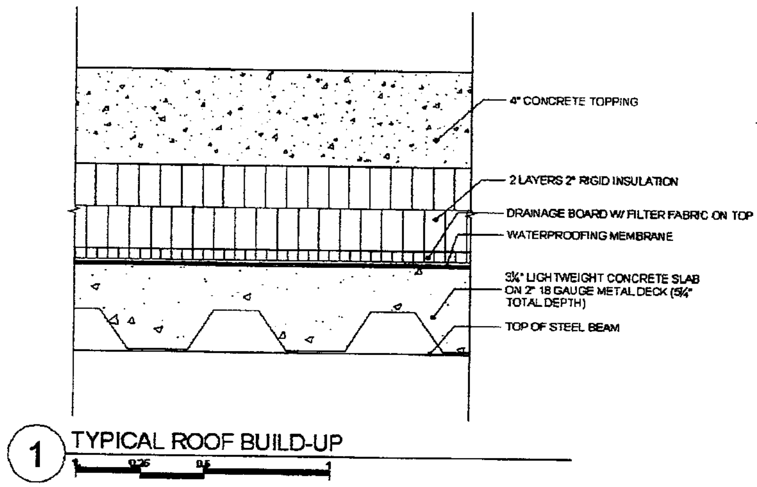


*Figure 21- As Built Roof Build-Up*

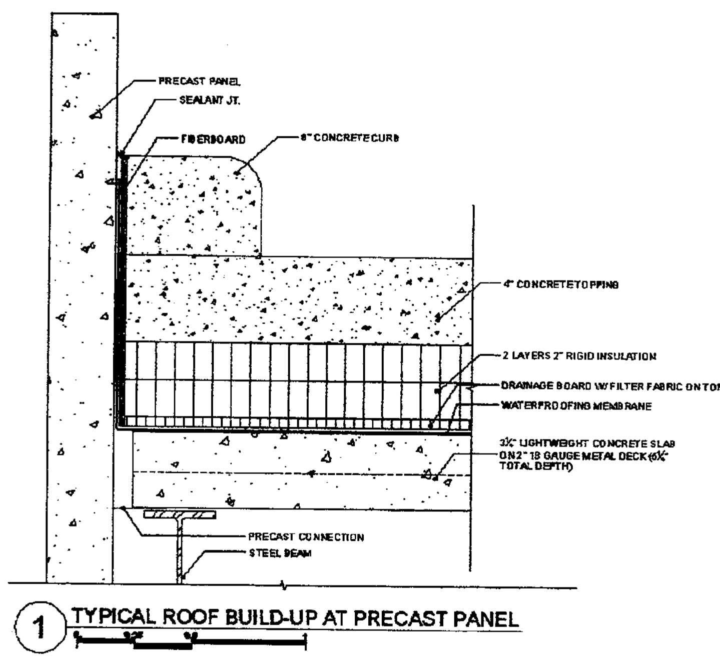


*Figure 22-Roof Build-Up as Observed by WJE at Precast Panel*

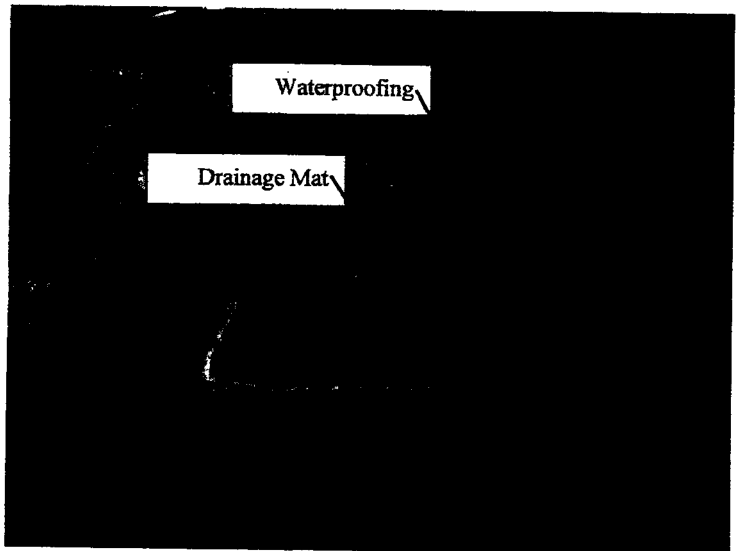


*Figure 23- Drainage Mat Terminating Before Face of Wall*

*Figure 24- Cracking and Displacement of CMU at South Bulkhead. Displaced CMU are in alignment with topping slab outside*

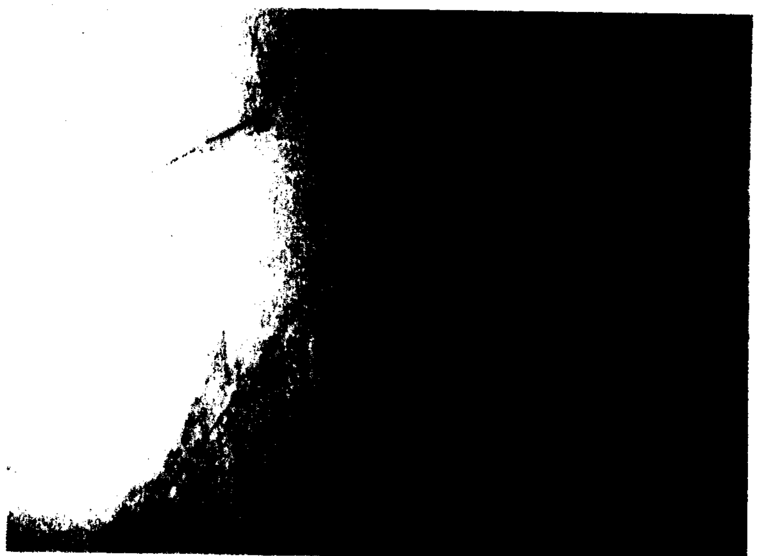

*Figure 25- Displaced CMU at South Bulkhead. Displaced CMU are in alignment with topping slab outside*

*Figure 26- Existing Repaired Cracks in the Northeast Stair Bulkhead Structure*

*Figure 27- Existing Repaired Cracks and new Cracks in the Northeast Stair Bulkhead Structure*

PLANT HARDWARE

2-3 RT-134 INSERT w/ 10M HAIRPIN

ERECTION HARDWARE

3-2 0"ø STD. WASHER

3-3 0"ø x 2" LG. BOLT

3-4 3"x4" KORDLATH HORSESHOE SHIMS

3-20 ? 5"x ?"x12" w/ 2-SLOTS

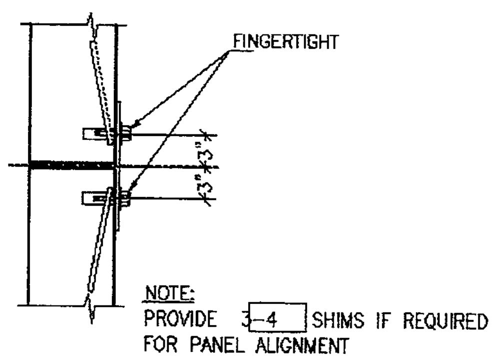


*Figure 28- Typical Precast Panel to Panel Connection*